Coastal Engineering Technical Note

BAFFLED BREAKWATER FOR LIMITED FETCH SITES

PURPOSE: To describe the Spud Point Marina breakwater, an innovative baffled-type breakwater designed to permit marina flushing while reducing short-period wave penetration.

BACKGROUND: Spud Point Marina breakwater is located in the northwestern part of Bodega Harbor, an enclosed bay on the California coast, about 60 miles north of San Francisco (Figure 1). Bodega Harbor is protected from the open Pacific Ocean by a rocky headland and peninsula, Bodega Head. A sandy peninsula across the southern part of Bodega Harbor, Doran Beach, separates it from the area of the open coast known as Bodega Bay. Access to the various marina facilities inside Bodega Harbor is provided through a jettied entrance and dredged navigational channels. Bodega Harbor is relatively shallow; the deepest parts are the dredged channels and turning basins (nominal project depth of -12 ft mean lower low water (mllw)). Spud Point Marina is operated by Sonoma County. The breakwater and access channel are maintained by the Corps of Engineers in addition to the entrance jetties and other dredged channels and basins shown in Figure 1.

The breakwater is a concrete pile-supported baffled structure consisting of prestressed concrete vertical piles and a cast-in-place concrete cap beam, with prestressed concrete baffle panels between the vertical support piles under the cap beam. Angled prestressed concrete batter piles along the marina side of the structure are embedded into the cap beam to form bents, giving increased lateral support. The top of the cap beam is located at an elevation of +8 ft mllw datum. The baffle panels extend from the bottom of the cap beam down to -1 ft mllw. Since the breakwater is open between support piles below the baffle panels, the structure allows a varying degree of circulation through the marina, depending on the stage of the tide.

Figure 2 is an oblique aerial photo of the marina and breakwater looking in an easterly direction. Spud Point is the promontory and sand flat on the right (south) side of the breakwater; the main Federal channel and narrower access channel also are visible.

Figure 3 shows the middle part of the breakwater at a relatively low tide from a viewpoint on the hillside west of the marina. The batter piles and openings below the baffle panels can be seen, as well as handrails and a fishing platform for public use.

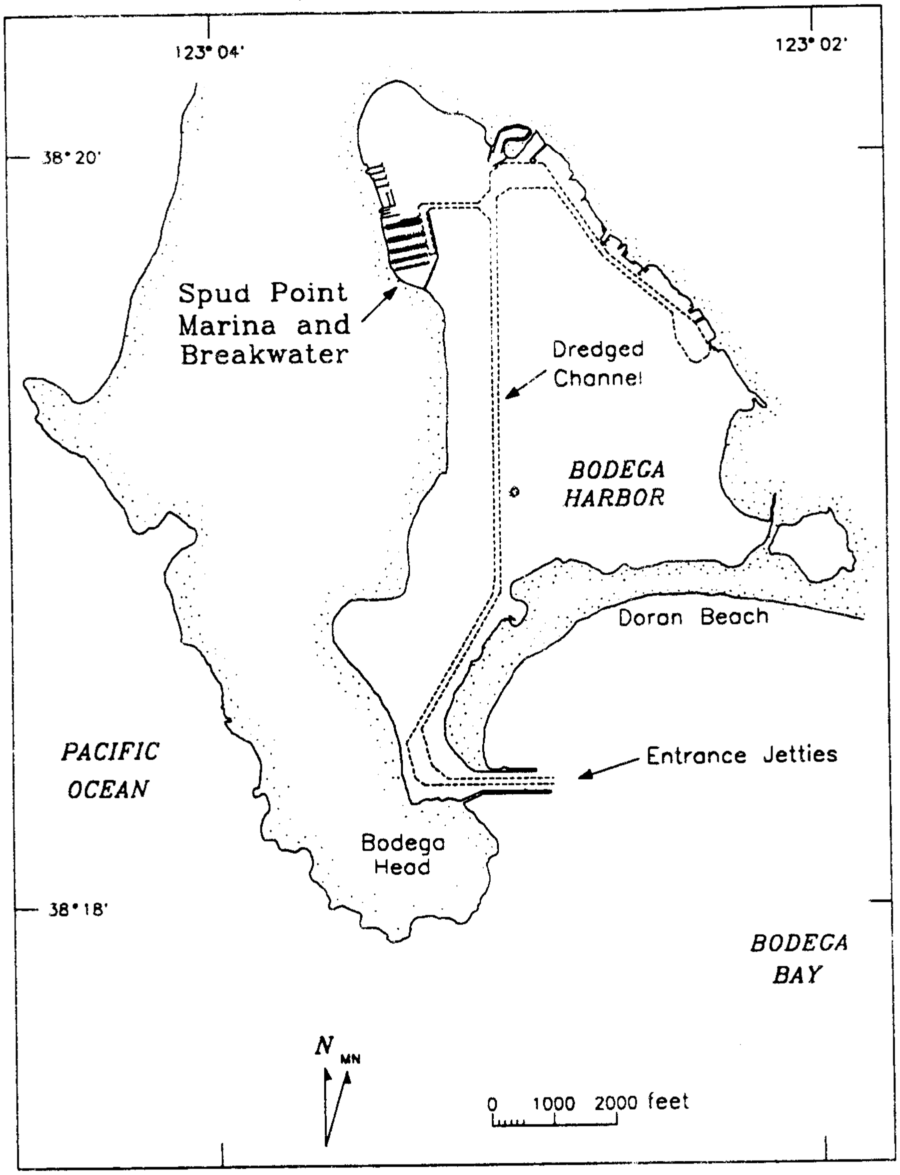

Figure 1. Site location map.

Figure 4 is a scaled site plan of the breakwater and vicinity. State Plane coordinates are shown. Dredged channels are indicated by dashed lines. The zero elevation mllw contour is shown as a solid line enclosing tidal flat zones indicated by hatching. Selected recent soundings referenced to mllw are shown to point out the large depth difference between the channel and the adjacent flat fronting the breakwater, where depths range from -5 to -1 ft mllw. North of the marina, depths are generally slightly greater than those in front of the breakwater.

Preliminary design of the breakwater was done by Moffatt & Nichol, Engineers, Long Beach, California, under contract to the US Army Engineer District (USAED), San Francisco (SPN). The breakwater was subsequently built by Sonoma County. Final design and construction supervision were also by Moffatt & Nichol. Because the as-built breakwater meets the criteria of the authorized Federal Project, maintenance responsiblity has since been assumed by SPN.

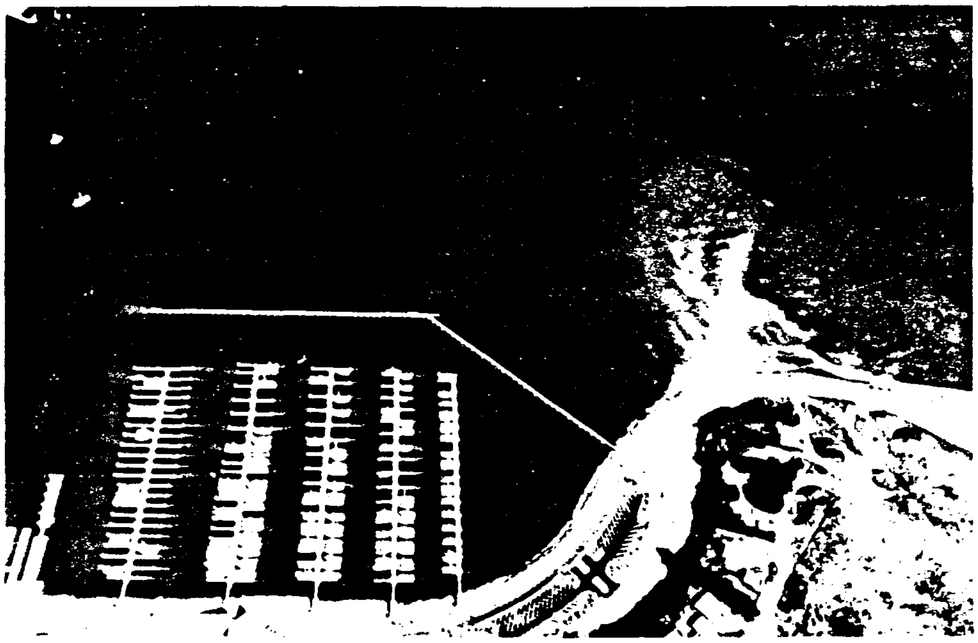

Figure 2. Oblique view of Spud Point Marina and breakwater looking east.

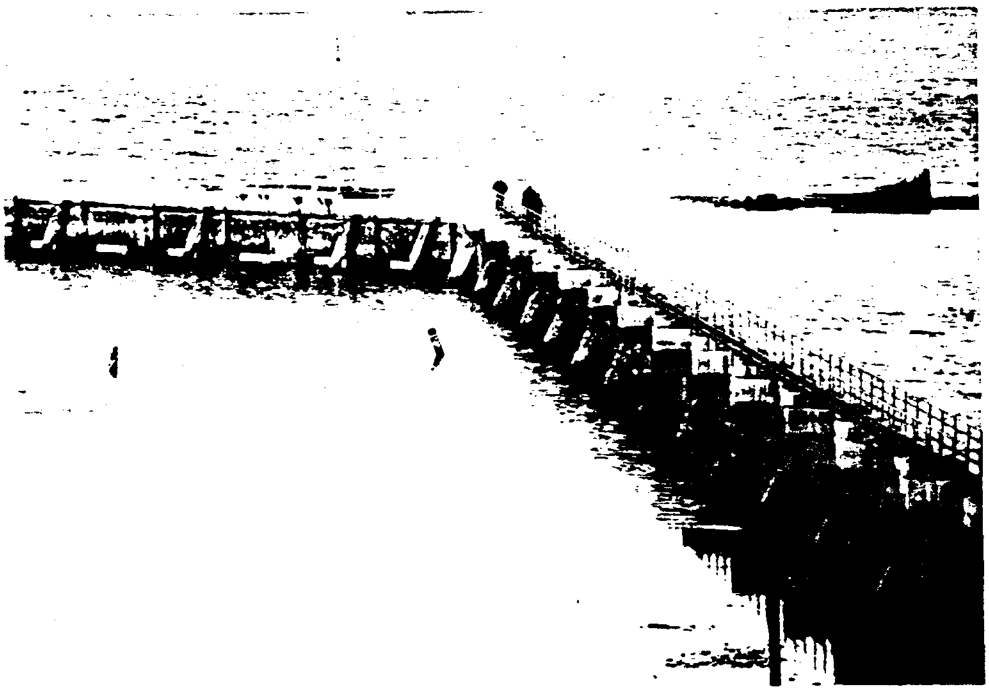

Figure 3. View of breakwater at an extrememly low tide.

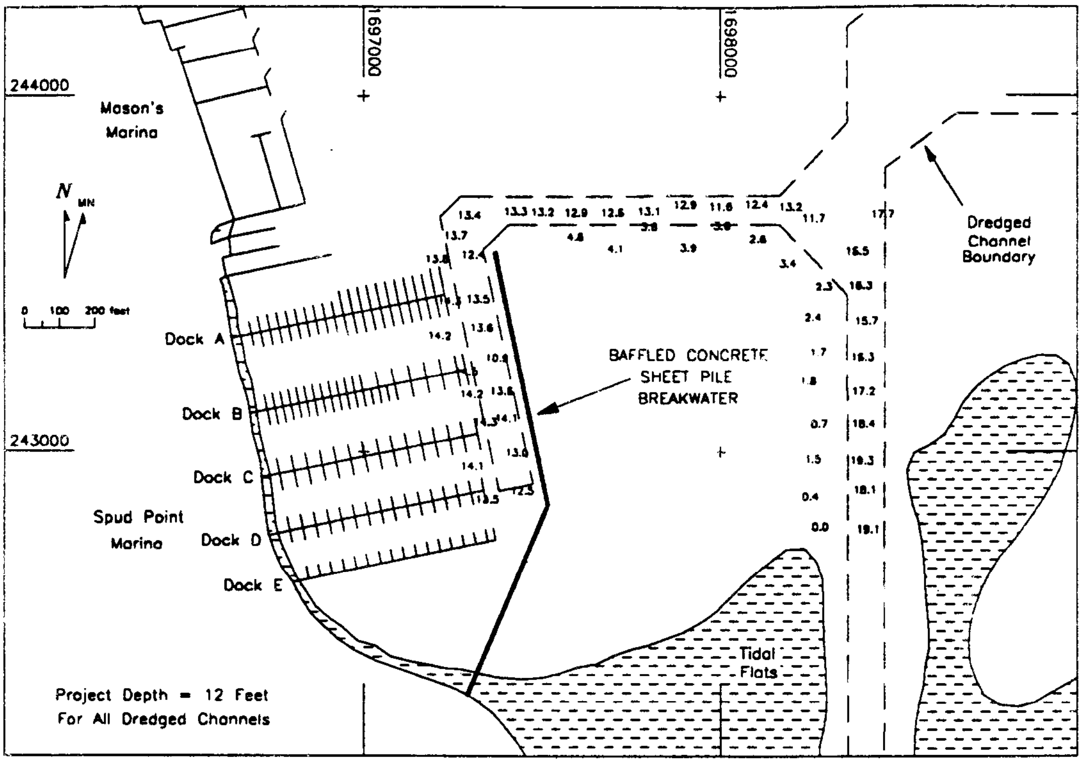

Figure 4. Detail map of marina and breakwater.

BREAKWATER DESIGN: The tidal range between mllw and mean higher high water (mhhw) is 4.1 ft at the north end of Bodega Harbor. The mhhw, 4.1 ft, and the highest estimated tide (het), 6.5 ft, were critical to design of the breakwater cap elevation.

No explicit consideration of current effects on the structure, such as scour potential or wave/current interaction, was included in the breakwater design. Qualitative information indicated that pre-existing currents were generally north-to-south at the proposed breakwater location. The structure's effects on currents and, consequently, on flushing and water quality in the marina were an important consideration in the decision to use a permeable baffled structure.

Design wave characteristics were determined using forecasting procedures found in the Coastal Engineering Research Center's (CERC's) *Shore Protection Manual* (SPM 1977). Pacific Ocean waves do not penetrate as far as Spud Point, so only locally-generated waves needed to be considered. Boat wakes within the marina were expected to be negligible, since speeds are restricted to 5 mph in such areas. A wind rose for seven years of record at the U.S. Coast Guard (USCG) station at Doran Point was available. Winds from northwesterly to southerly directions were considered unimportant for wave generation, due to sheltering of the site by Bodega Head peninsula, and lack of long fetches in those directions. From a review of the wind rose versus fetch orientation, fetch length, and water depths, two critical fetches were selected for calculation of wave forecasts (see Figure 5). The SPM shallow-water forecasting curves were used assuming a stillwater level (swl) of 4.1 ft (the mhhw elevation for northern Bodega Harbor). Wave characteristics under het conditions were not forecast.

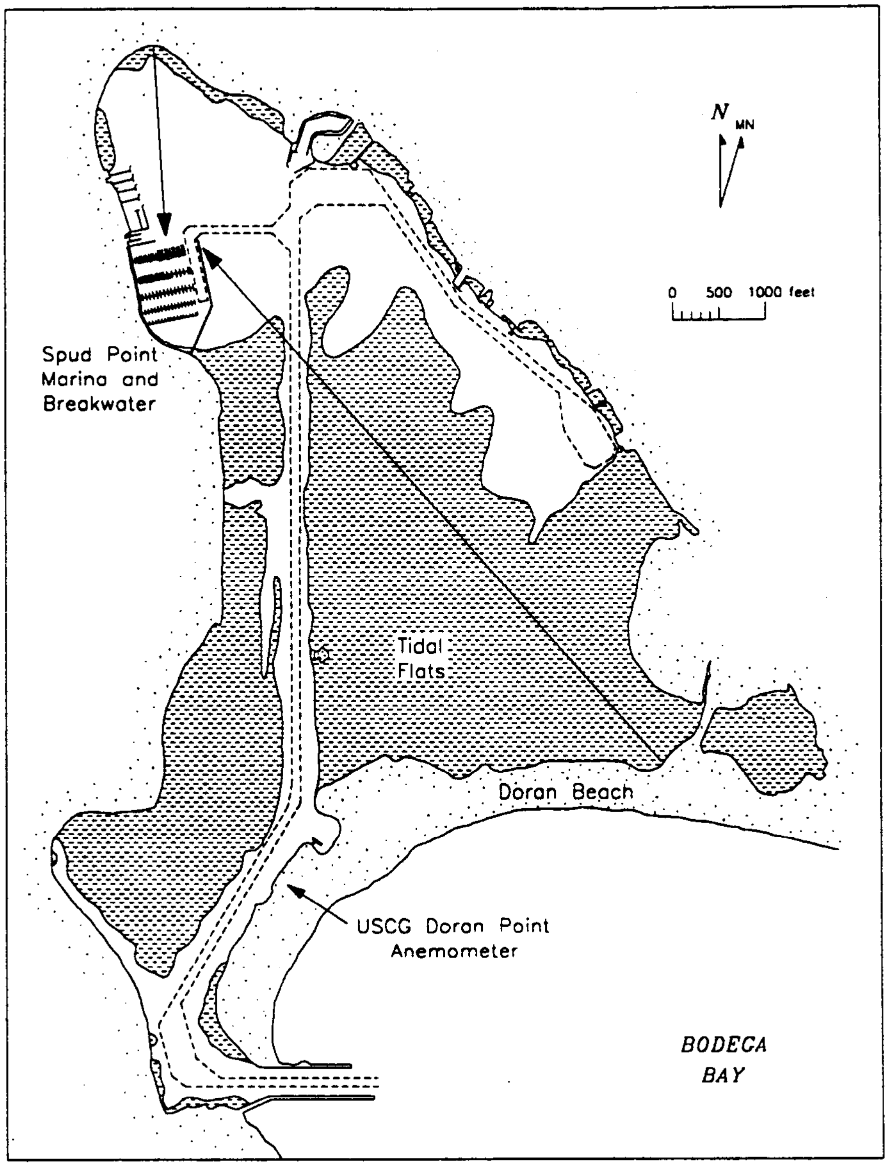

Figure 5. Critical wave generation fetches for Spud Point site.

It was determined that no breakwater would be needed for the north side of the marina. The wave forecast for the southeasterly fetch resulted in significant wave height and period, H_s and T_s, of 1.5 ft and 2.5 sec, respectively, giving a design wave height, H_1 of 2.5 ft. The SPM recommends use of H_1, the height of the average of highest 1 percent of all waves (= 1.67 H_s), as the design wave height for rigid structures.

Selection of the Spud Point site and recommended breakwater design involved many other considerations, including costs over a 50-year design life span and environmental impact. Configuration and maintenance of dredged channels and dredged material disposal options played an important part in selection of the recommended plan. Bottom sediments and bathymetry were considered with respect to dredging of the marina and access channels, and foundation conditions for the breakwater. The Corps' General Design Memorandum (GDM) (USAED, San Francisco 1981) describes alternative plans and covers the selection process in detail.

The concrete structure includes baffle panels, vertical and batter support piles, and a continuous cap beam. In the most seaward part of the breakwater, every vertical pile forms a bent with a tilted batter pile on the marina side. For the rest of the breakwater, bents alternate with vertical intermediate piles. Baffle panels were designed as simple beams, supported by the vertical piles. Their design considered bending stresses due to wave loads and panel dead load. The batter piles were designed for axial load due to wave loads and bending stresses due to dead load. The vertical bent piles were designed for axial load due to dead and wave loads and bending stress due to wave loads. The intermediate piles are similar to the vertical bent piles. The cast-in-place cap was designed as a continous beam for vertical loads supported by the bents and intermediate piles, and for horizontal loads supported by the bents alone. The GDM states that wave forces were calculated in accordance with EM 1110-2-2904, "Design of Breakwaters and Jetties" (US Army Corps of Engineers 1986), as applicable to non-breaking, breaking, and broken waves. Breaking conditions were determined on the basis that the average ratio of the depth of water in which a wave breaks to the wave height at the point of breaking is approximately 1.3. Wave forces were calculated by the Sainflou method for non-breaking waves, and by Minikin's method for breaking waves, both of which are presented in EM 1110-2-2904.

The crest height of the breakwater (top of cap elevation) was arrived at using methods detailed in EM 1110-2-2904 for the case of non-breaking waves forming a standing wave (clapotis) due to reflection off the wall. The design wave height, 2.5 ft, was used along with a swl (= mhhw elevation) of 4.1 ft plus a wind setup of 0.4 ft in an equation which EM 1110-2-2904 refers to as the modified Sainflou method or the Miche-Rundgren method. The method gives the superelevation (above the swl) of the mean level of the clapotis. The crest of the clapotis was calculated to occur at about +7.7 ft mllw, determined as this mean level plus the design wave height. The cap elevation was consequently set at +8.0 ft mllw. For the het swl of +6.5 ft mllw, the calculated clapotis crest elevation was +10.2 ft, an overtopping condition. Since het conditions were considered an infrequent occurrence, and the distance between the breakwater and the nearest berths was considered likely to be sufficient to result in dissipation of overtopped waves, the +8.0 ft cap elevation was considered to be adequate.

As previously described, the baffled breakwater uses baffle panels which extend only part of the way from the cap to the harbor bottom. Lowering the panel bottom elevation decreases the wave transmission, but also decreases the flushing potential of the marina. The design approach was to find the highest panel bottom elevation possible, given the allowable wave transmission criteria. The GDM states that normal criteria for acceptable maximum wave heights in berthing areas limit maximum wave heights to 1.5 ft. Since there is no published Corps design guidance for this type of breakwater, results presented by Wiegel (1960, 1964) in his landmark book, *Oceanographical Engineering*, were used. Wiegel developed a nomograph from a first-order transmission expression based on wave power for the case of an infinitely thin, rigid barrier extending from above the water surface to some distance below the water surface. The theoretical approach assumes monochromatic waves over a flat bottom and does not consider the effects of supporting piles. A detailed description of the theoretical development and laboratory tests performed for comparison with the theory's predictions are given in an earlier journal article (Wiegel 1960). Figure 6 shows the version of Wiegel's nomograph given in the US Navy's Coastal Protection Design Manual (US Navy 1982). The nomograph relates the transmission coefficient, K_t, (ratio of transmitted wave height to incident wave height) to the ratio of the baffle panel penetration depth to the stillwater depth, h/d_s, for a range of relative depths. The relative depth is the local ratio of depth to wavelength, d/L,

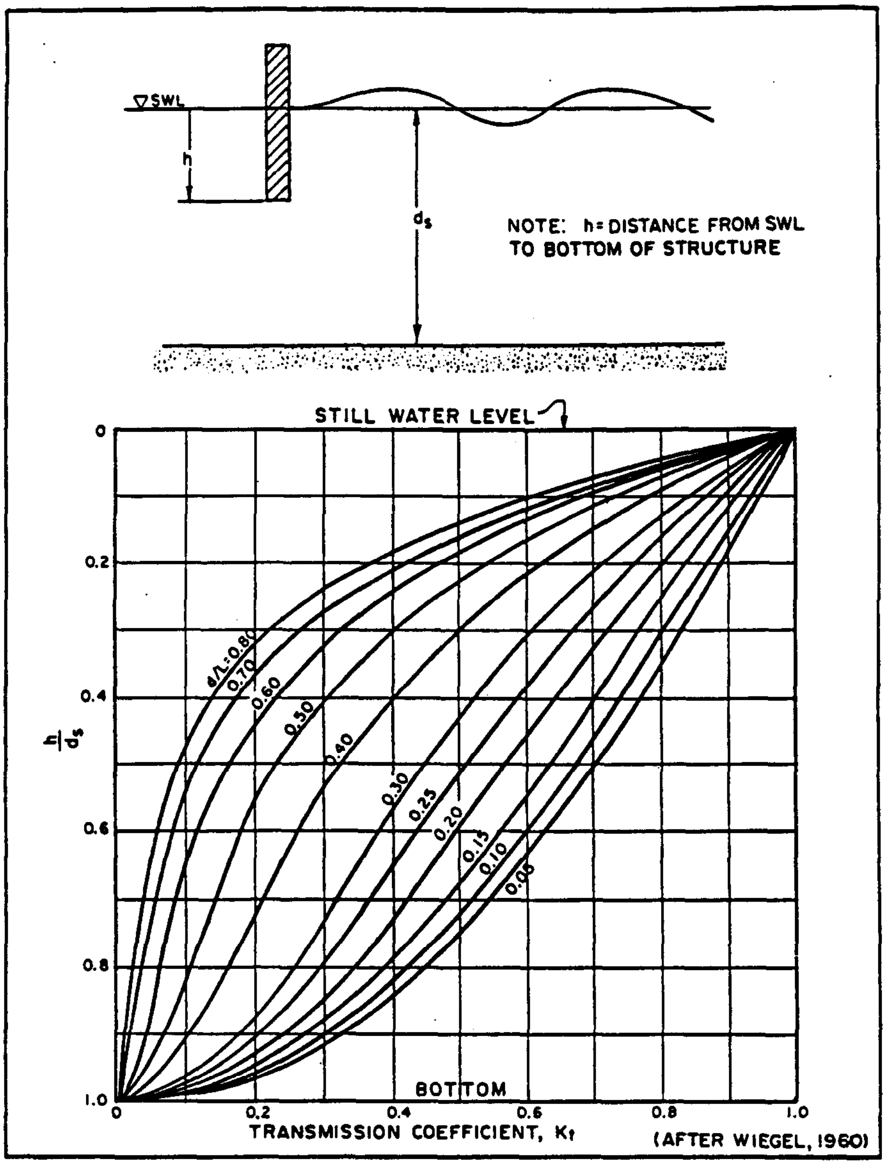

Figure 6. Nomograph for wave transmission of baffled structure.

occurring at the structure. For example, one might compute the deepwater wavelength, L_o, using the design wave period, then compute an equivalent deepwater relative depth, d_s/L_o, (using the stillwater depth at the structure) then find local relative depth, d/L, using Table C-1 of the SPM (which relates d/L and d/L_o). Given a design swl, mudline elevation on the structure, and incident wave height, the nomograph can be used to determine the required baffle bottom elevation corresponding to a transmitted wave height criterion. The GDM states that with the bottom of baffle panels set at -1.0 ft mllw, the transmitted wave heights would be about 1 ft under design conditions for the mhhw and het swls (K_t of about 0.4 in Figure 6), and lower than 1 ft for lower tidal stages. Although not stated in the GDM, apparently the mudline elevation on the structure was assumed for transmission calculations to be at about -7 ft mllw, which was also assumed in the final design by Moffatt & Nichol.

POST-CONSTRUCTION MONITORING OF BREAKWATER PERFORMANCE: A field monitoring study of the Spud Point Marina breakwater was conducted by CERC in conjunction with SPN as part of the Monitoring Completed Coastal Projects (MCCP) program (Lott 1991). A field study of wave transmission was conducted using boat wakes and pressure sensors to measure the generated waves. USCG lifeboats from the Doran Point station were provided for generation of wakes. Soundings of potential scour zones and a side-scan sonar survey were made. Circulation through the breakwater and marina was measured, and the breakwater was examined for structural integrity. Flushing performance appeared to be satisfactory. No evidence of scour or structural displacement was found.

At higher tide stages during the boat wake experiment, some generated waves were able to penetrate the marina, but these same large wakes were producing unacceptable boat motions in other marinas on the opposite side of Bodega Harbor, forcing a pause in the experiment to wait for lower tides later in the day. The transmitted wave heights were too small to be quantified from the pressure records, and were well below design criteria. Later, at the lower tide stages, generated waves were highly dissipated as they crossed the shallow region fronting the breakwater.

Although the wave attenuation was not quantified by the boat wake experiment, the difficulty experienced in producing transmitted waves is evidence that the breakwater is providing significant wave attenuation. The marina operator reports that the breakwater performance has been very satisfactory, and that only the largest tenant boat (65-ft-long) produces a wake large enough to penetrate the breakwater and cause rolling of the docked boats. Figure 7 is convincing evidence of the breakwater's wind wave attenuation performance under severe conditions. Even with overtopping, water inside the marina remains calm.

Although the transmitted wave heights were small during the experiment, the significant rolling of some of the boats due to the largest wakes suggests that parameters other than wave <u>height</u> may be of interest for wake or wave transmission criteria. It is unknown whether wind waves of similar height would have caused the rolling. Boats were docked so that they were broadside to the breakwater. The baffled type of breakwater may reduce vertical water particle motions and surface disturbances, yet allow appreciable horizontal motion to pass through the breakwater in the lower part of the water column. This emphasizes that for some protected breakwaters, protection against wakes may govern the design more than protection against wind waves.

SUMMARY: Based on the experimental, anecdotal, and photographic evidence, the Spud Point Marina baffled breakwater appears to be performing acceptably and Wiegel's nomograph seems suitable for its design. Designers of breakwaters in low-energy environments where flushing performance is critical should consider a baffled structure.

Since the relative contribution of site-specific conditions (particularly the shallow flats fronting the breakwater) to successful performance of the Spud Point breakwater is unknown, care should be taken when extending the use of the nomograph and the baffled structure beyond preliminary conceptual studies. Physical model testing may be needed to more confidently predict wind wave and boat wake attenuation where costs or risks are high, until a more comprehensive evaluation of the theory and performance of baffled breakwaters has been completed.

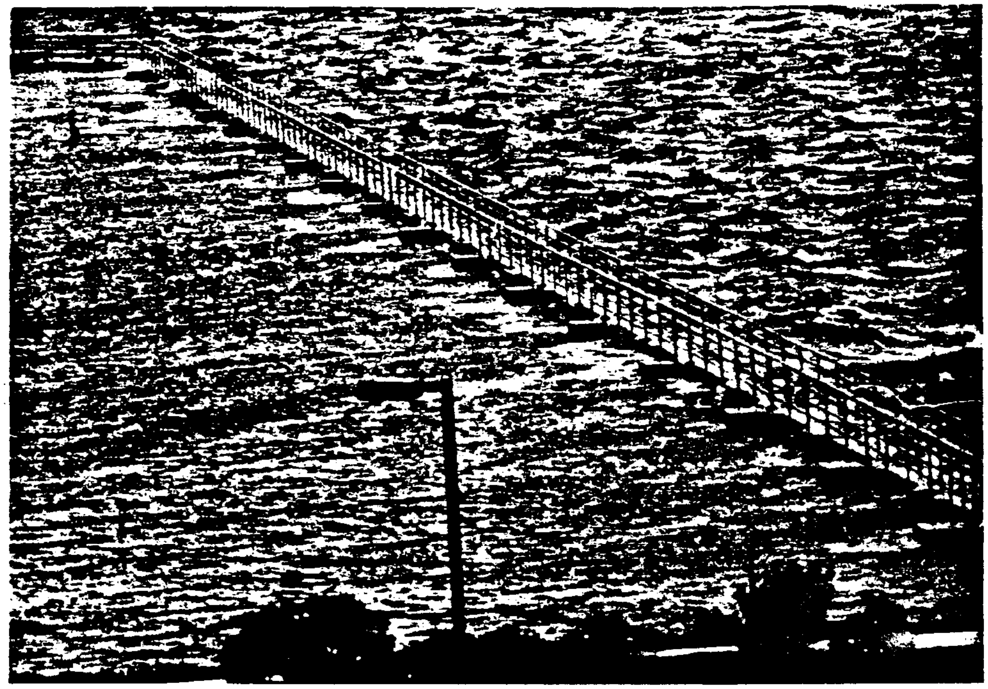

Figure 7a. View of shoreward leg of breakwater at high tide during storm.

Figure 7b. Marina inside shoreward leg remains calm despite overtopping.

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